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WIND LOADS ON TRUSS BRIDGES

by John M. Biggs

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WIND LOADS ON TRUSS BRIDGES

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Design specifications for wind loadings on railroad and highway bridges have remained essentially unchanged for over half a century. This is true in spite of the fact that facilities capable of experimental determination of these loads have been developed during that time.

In recent years data have become available through research concerning the aerodynamic stability of suspension bridges. However, wind loads obtained in tests of suspension bridge cross sections are of limited value because the proportions are quite different than those encountered in other types of bridges.

Presented in this paper are the results of wind tunnel tests on two scale models of a typical double track railroad bridge. By modifying one of these basic models, other data have been obtained for highway bridges and for various truss spacings. It is not intended to present a study of basic aerodynamic theory. However, since the prototype of the models is a typical simple span truss bridge, these test results should be useful to the designer of such structures.

The prototype is a double track, open deck railroad bridge of the subdivided Warren type having a span of 350 ft. and a depth of 49.2 ft. All truss members are fabricated of plates and angles to form box or I-sections. The floor system is of the conventional floor beam and stringer type.

The first of the two models tested was constructed of wood to a scale of 1:120 (see Fig. 1). As much detail as was possible with this material and scale was included. U or H-sections on the prototype were simulated by routing one or two sides respectively of solid rectangular cross sections. Members having lacing bars on one or two sides were represented by rectangular pieces in which circular holes had been drilled leaving the same ratio of net area to enclosed area as occurred on the prototype.

The second model was constructed of brass to a scale of 1:40 (see Fig. 2). It differs from the first model in that it includes only eight interior panels or 2/5 of the span. The details of the prototype were carefully reproduced in an attempt to ensure the same aerodynamic properties. All gusset plates, lacing angles, batten plates, and all projections on the individual members were reproduced to scale.

All the time the models were constructed it was thought that the drag coefficients might differ for the following reasons:

1) The greater detail in the brass model,
2) The different aspect ratio (length divided by width - 7.1 and 2.8 for the wood and brass models respectively) which is known to affect the drag coefficient of solid plates, and
3) The end effects which might be present on the brass model.

The data indicated, however, very close agreement between the wind pressures obtained on the two models. In Fig. 3 is plotted the wind pressure on

each model against angle of attack, α, which is the angle of wind direction measured from a horizontal plane.

It was desirable to obtain more basic data which might be applicable to bridges other than the prototype. To this end the brass model was constructed so that its main parts could be assembled and tested in any combination (see Fig. 4). In this manner the contributions of the various parts to the total wind force can be determined and a general method for computing wind loads devised.

The following sequence of testing was adopted: (1) a single truss, (2) both trusses without the floor system or lateral bracing, but spaced by small rods having little resistance to the air stream, (3) both trusses plus the floor system and the bottom lateral bracing, and (4) the complete bridge (see Fig. 5). A second floor system having only two stringers and a width corresponding to a single track railroad bridge was also constructed. This floor together with narrow lateral bracing systems was combined with the two trusses and tested in the same sequence to give results for a single track bridge. By replacing tracks and ties with a solid plywood slab with curbs, a reasonable facsimile of a highway bridge was obtained. It was expedient to use the same floor framing and lateral systems for both railroad and highway bridges. Typical road widths were therefore not possible. The roadways tested were 14.6 ft. and 26.2 ft. at full scale including curbs. The latter is approximately a two-lane width.

The tests were conducted according to standard wind tunnel procedures at wind velocities of 50, 60, and 70 mph. The aerodynamic coefficients obtained at all three speeds were in agreement.

Test Results

Following the sequence outlined above the first test was run on a single truss completely removed from the rest of the structure. The horizontal drag forces obtained at three wind velocities for various angles of attack are shown in Fig. 6. In the formula,

$$H = A_T \, C_D \, \rho \, \frac{V^2}{2} ,$$

H is defined as the total force on the model in a direction normal to the truss itself, A_T is the truss area projected in side elevation including gusset plates and other details, V is the wind velocity, and ρ is the density of air.

In the case of rounded solid bodies immersed in an air stream the drag coefficient, C_D, varies with velocity below a certain critical value. Although this is not generally true of sharp-edged bodies such as the members of which a bridge is composed, it was thought necessary to verify the existence of a constant coefficient for the model tested. An inspection of Fig. 6 reveals that the horizontal force is essentially proportional to the velocity squared and the drag coefficient for this range of velocities is therefore constant.

Using a wind velocity of 70 mph the drag coefficient at zero angle of attack is computed as follows:

$$C_D = \frac{H}{A_T \, \rho \, \frac{V^2}{2}} = \frac{28.9 \times 144}{203.1 \times 12.543} = 1.63$$

The value of C_D is known to vary with "solidity" which is defined as the ratio of the solid area of the members to the total area occupied by the truss

in its vertical plane. This truss has a solidity of 0.25 which is fairly typical. Therefore, the drag coefficient given is applicable to most simple span bridge trusses.

The two values of wind pressure normally used in bridge design are 30 and 50 lbs. per sq. ft. It is of interest to note that according to the tests on a single truss the wind velocities corresponding to these pressures are 84.9 and 109.5 mph respectively.

In order that the data might be presented in a manner consistent with design specifications a new factor, C_A, is introduced into the basic equation:

$$H = (C_A A_T) C_D \rho \frac{V^2}{2}$$

In this equation $C_D = 1.63$, A_T is the elevation area of one truss, and C_A is the area coefficient. For any combination of the components of the bridge or any wind direction other than horizontal, C_A is the factor by which the area of one truss is multiplied to give the effective area under those particular conditions. Thus for a single truss subjected to a horizontal wind C_A is unity. It should be recognized that C_A is merely a measure of the variation in total horizontal force on the model as angle of attack is varied and as other component parts are added to the single truss.

In Fig. 7 is shown the variation in C_A with angle of attack for the single truss. Positive angles indicate a wind component upward on the bridge.

Fig. 7 also shows values of the area coefficient for various spacings of two parallel trusses. The extreme spacings tested correspond to 13.3 ft. and 60 ft. at full scale. The amount by which C_A exceeds unity represents the contribution of the leeward truss to the total horizontal force as well as the effect of variation in angle of attack. C_A, or the total horizontal force, increases with truss spacing. If the trusses were very close together the leeward members would be almost completely shielded by their windward mates. As the spacing is increased the disturbance caused by the windward truss begins to disappear before the leeward truss is reached. At an infinite spacing the area coefficient would be 2.0. At angles of attack other than zero the shielding effects are reduced and the load on the leeward truss is increased.

As may be observed in Fig. 7, the load on the leeward truss varies considerably as the truss spacing is changed. In spite of this fact the effect of truss spacing is ignored by most specifications.

There is very little meteorological data from which can be determined the maximum angle of attack for design purposes. The maximum angle depends to some extent upon the site of the structure but a reasonable range is ± 15 degrees. Within this range the maximum area coefficient obtained for the 18-inch model truss spacing (60 ft. at full scale) is 1.73. This corresponds to a load on the leeward truss equal to 77% of that on the windward truss which is considerably greater than the value of 50% given by some specifications.

Figs. 8 and 9 show values of C_A for all configurations of the four bridge types. When the floor system is added to the two trusses the effect varies with bridge type and with angle of attack. At most points the addition of the floor system results in a decrease of total horizontal force below that which acted on two trusses alone. At no point does the floor cause an appreciable increase in load. At the angle of attack corresponding to the maximum horizontal forces on the complete bridges (-15° for the single track railroad bridge and $+15^\circ$ for other types) the railroad floor systems cause a slight increase in load whereas the highway floors cause a considerable decrease.

This surprising lack of drag on the floor may be explained by the fact that the region shielded by the windward chord has a vertical dimension greater than the depth of the chord itself and may, in fact, contain the entire leading edge of the floor system. The only horizontal force on the floor is therefore the friction drag on its top and bottom surfaces. The floor system almost entirely shields the leeward chord and a reduction in drag results therefrom. If this reduction exceeds the friction drag on the floor the net effect of the addition of the floor system is a decrease in the total horizontal wind force.

The situation described above varies with angle of attack. As would be expected the addition of the floor generally tends to increase the total horizontal load when the wind direction is down on the bridge (negative α) and to decrease the load when the wind has an upward component (see Figs. 8 and 9). The relative position of the floor with respect to the chord is also important. As indicated by later tests the wind load is greater if the floor position is above or below that in the prototype.

When the top lateral system is added to form the complete bridge the effect is invariably to decrease the total horizontal force (see Figs. 8 and 9). For the wide bridges the decrease is very slight. In the case of the narrow bridges the decrease is somewhat larger.

Within the probable range of angle of attack ($\pm 15^0$) the maximum values of C_A for the four complete bridge types are as follows:

Double track RR	1.69
Single track RR	1.47
Two-lane Hwy	1.52
Narrow Hwy	1.42

It should be recalled that C_A is the factor by which the horizontal force on the windward truss at $\alpha = 0$ is multiplied in order to obtain the total horizontal force for the complete bridge. An angle of yaw would increase these values slightly (see below).

Effect of Floor Position

The next investigation involves the effect of the position of the floor system with respect to the trusses. In Fig. 10 the area coefficient of the wider bridge is plotted with four floor positions other than the "normal" position or that which existed in the prototype. In positions I and II the floor system is suspended below the trusses, whereas in positions III and IV the bottom of the floor beam is above the bottom of the truss chord. Fig. 2(b) shows the model with the floor in position IV.

In Table I are given the change in area coefficients due to the presence of the floor (ΔC_A) for each position tested at an angle of attack corresponding to the maximum horizontal force. Also tabulated are the effective elevation areas of the floor which are equal to ΔC_A times the elevation area of one truss (203.1 sq. ins.). These areas are compared with the unshielded area of the floor as seen in the side elevation, a factor which is considered in most specifications. The unshielded areas in positions I and II are the full elevation area of the floor. The effective floor area is always less than the unshielded area and has very little or no relation to it. The effective area is related, however, to the distance from the bottom chord. It is apparently incorrect to compute the load contribution of the floor system as a function of unshielded elevation area.

Lift and Overturning Moment

Two factors which are of importance to the designer but which are not included in specifications are the lift and overturning moment due to wind forces. In Fig. 11 the lift or vertical force on the bridges with normal floor position is plotted as a ratio to the horizontal force, positive lift being upward on the bridge. It will be observed that the highway bridges are subject to larger lift forces than are the railroad bridges and also that the magnitude of the lift ratio increases with width of bridge. The maximum vertical force on the two-lane highway bridge within the probable range of α is six tenths of the horizontal force.

Plotted in Fig. 12 is the effective moment arm as a ratio to truss depth. The moment is computed about the leeward bottom chord and the effective arm is defined as that moment divided by H. Thus this moment arm is comparable to that normally used in design when lift is neglected. Except for the two-lane highway bridge the moment arm is not far from the truss centroid. In this exceptional case the maximum effective moment arm (at $\alpha = +15^{\circ}$) is 0.72 of the truss depth. This indicates that the maximum overturning moment is about 50% greater than that normally used in design. Wider bridges would have greater values of lift and overturning moment.

When the wind is horizontal ($\alpha = 0$) the lift forces are not appreciable and the overturning moment is essentially equal to that caused by the horizontal load applied at the tress centroid. The larger lift forces are due primarily to an inclination of the wind direction and the variation in the overturning moment is due to the presence of these lift forces.

Some previous investigators have found much greater lift forces and overturning moments than those recorded in these tests. The disagreement is probably the result of two variables: (1) Truss solidity, and (2) the ratio of bridge width to truss depth. The models herein described are typical of simple span truss bridges and the results obtained are therefore applicable to this type of structure.

Effect of Yaw

Tests were run on the small scale wooden model (see Fig. 1) to determine the effect of yaw or variation in the angle of wind direction in the horizontal plane. Fig. 13 shows the increase due to yaw of the horizontal force at zero angle of attack. The maximum increase is 9% and occurs at $\psi = 20^{\circ}$. This increase is somewhat less for angles of attack other than zero. Inconclusive test results indicate that the increase at the angle of attack corresponding to maximum horizontal force ($\pm 15^{\circ}$) is about 5%.

Forces on Loaded Structure

In order to determine the effect of a train on the horizontal force, a string of scale model box cars were placed on one track of the model bridge. The resulting increase corresponds to a force of 238 lbs. per ft. on the prototype at a wind velocity of 85 mph which corresponds to 30 lbs. per sq. ft. on a single truss. The design value usually specified is 300 lbs. per ft.

Comparison with Specifications

Based upon the results described the total horizontal force may be expressed as follows:

$$H = C_D\, C_\psi\, C_A\, A_T\, \rho \frac{V^2}{2}$$

in which, C_D = drag coefficient of one truss = 1.63

C_ψ = yaw coefficient = 1.05

C_A = area coefficient

A_T = elevation area of one truss (ft^2)

$\rho \frac{V^2}{2}$ = 0.00256 V^2 (V in mph)

or, $H = 0.00438\, C_A\, A_T\, V^2$ lbs. per sq. ft.

A comparison of this formula with V = 109.5 mph to present specifications at 50 lbs. per sq. ft. is given in Table II. The loads computed are for the brass sectional model. The wind velocity selected is that which produces 50 lbs. per sq. ft. on a single truss. Specifications do not indicate the wind velocity intended. C_A is taken as the maximum value in the probable range of α(see above). The total horizontal load given by the specifications compares favorably with the experimental results and is always conservative. However, the manner by which the loads are computed is questionable.

The A.A.S.H.O. specifies 50 lbs. per sq. ft. on 1 1/2 times the projected elevation area including trusses and floor system. By this method the load on the leeward truss is underestimated for normal truss spacings (see Fig. 7). In effect the load on the floor is computed as the pressure multiplied by 1 1/2 times the unshielded floor area - a very unrealistic procedure (see Table I).

The A.R.E.A. specifies 50 lbs. per sq. ft. on the total projected elevation area plus the area of the leeward truss which is not shielded (as seen in elevation)by the floor. Regarding the trusses this procedure is conservative because the shielding of the leeward truss by its windward mate is entirely neglected. The effective area of the floor implied by this specification is the unshielded floor area minus the shielded portion of the leeward truss. By this method the effect of the floor agrees fairly well with the experimental results.

The tests described in this paper do not include a sufficient number of bridge types to warrant specific recommendations regarding design specifications for wind loads. However, it is apparent that present specifications could be improved by including the effect of truss spacing. Further research is required to develop a more realistic approach to the determination of the contribution of the floor system to the total wind load.

Conclusions

The following conclusions apply only to simple span truss bridges with proportions similar to those of the model tested:

(1) Wind pressures of 30 and 50 lbs. per sq. ft. on a single truss completely separate from other parts of the structure correspond to wind velocities of approximately 85 and 110 mph respectively.

(2) The spacing of trusses has considerable effect on wind loads. This fact is ignored by specifications. As the spacing increases from 13.3 ft. to 60 ft. the load on the leeward truss varies from 40% to 77% of that on the windward truss (see Fig. 7).

(3) The effect of the floor system on total horizontal load is generally small and bears little relation to the unshielded elevation area, a quantity which is considered by most specifications. The effect does, however, increase with the vertical distance from the floor to the truss chord.

(4) Lift and the resulting overturning moment are small on railroad bridges but may be of considerable importance on wide highway spans. On the relatively narrow two-lane bridge the overturning moment is 50% greater than that normally used in design.

(5) In order to achieve accurate wind tunnel tests it is not necessary to construct large-scale bridge models which include minute details of fabrication.

The tests described in this paper are a by-product of research done for the Corps of Engineers, U.S. Army, and were conducted in the Wright Brothers Wind Tunnel at M.I.T. under the supervision of John M. Cord and Warren B. Delano of the Department of Civil and Sanitary Engineering.

November 21, 1952

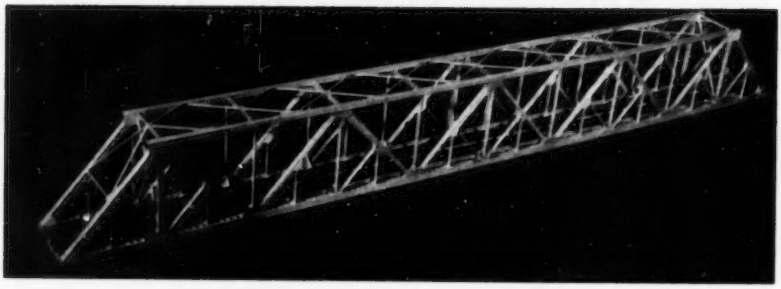
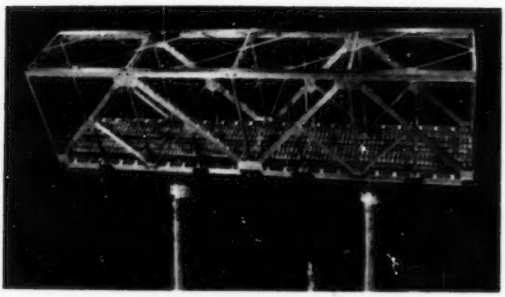
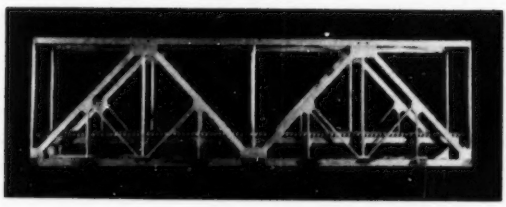


FIG. 1. Wood Model of Double Track Railroad Bridge.
Scale: 1 : 120



(a) Normal Floor Position



(b) Floor Position IV

FIG. 2. Brass Model of Double Track Railroad Bridge with Two Floor Positions.
Scale: 1 : 40

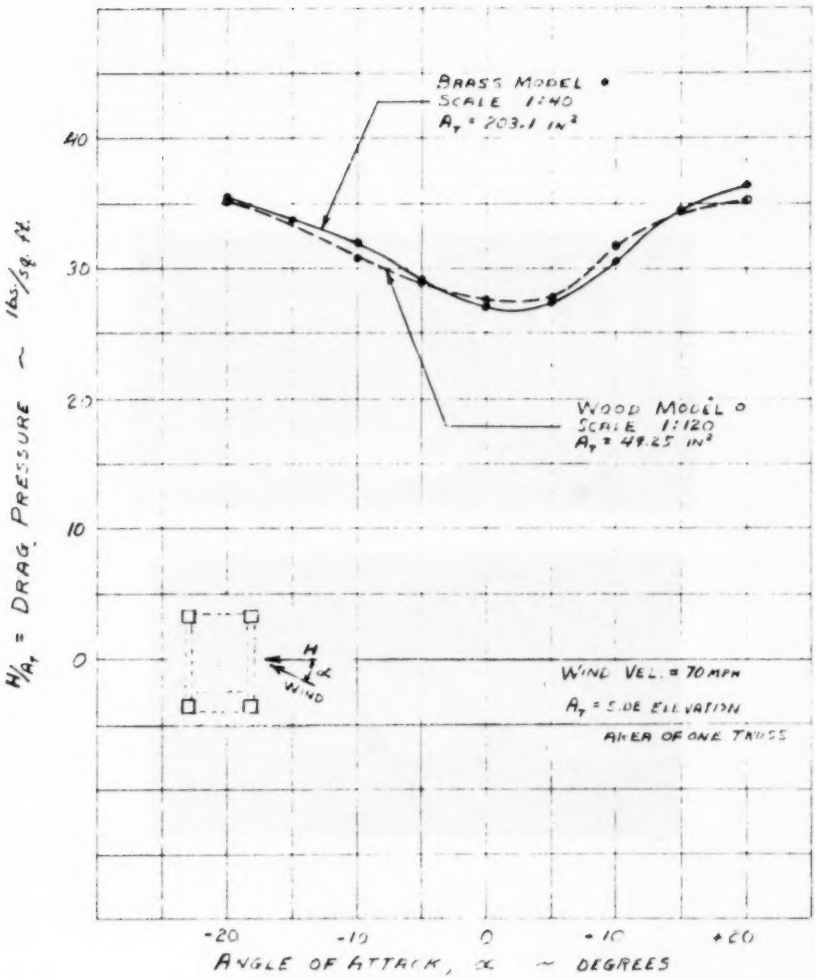


FIG. 3. Comparison of Drag Pressures on Two Models of Double Track RR Bridge.

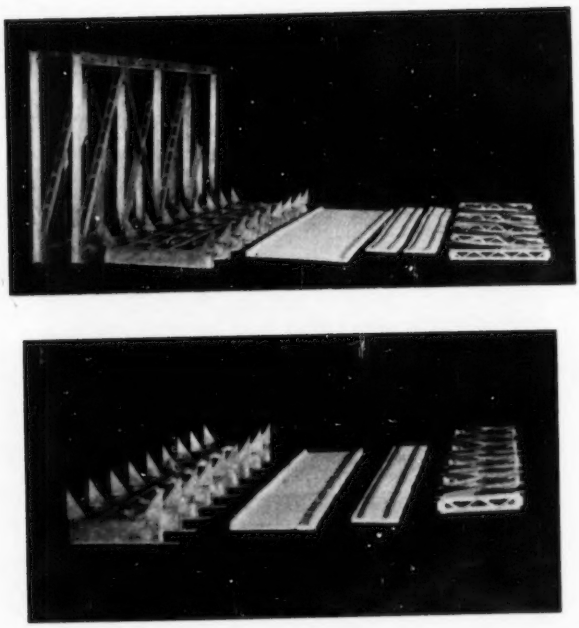


FIG. 4. Test Components.

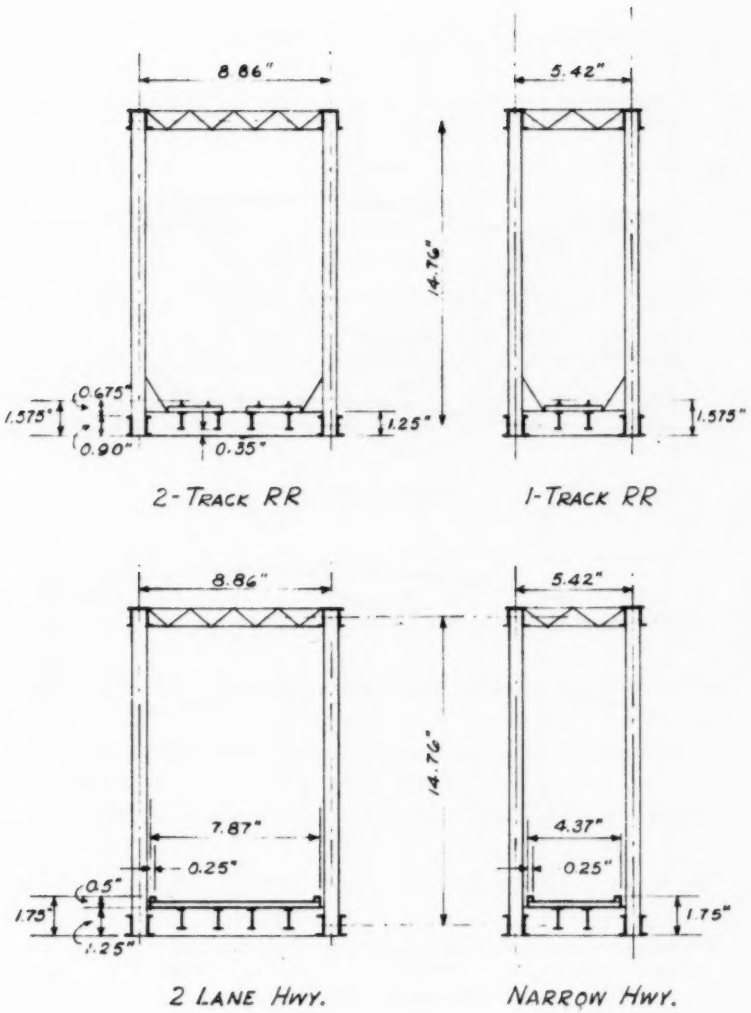


FIG. 5. Cross Sections of the Four Model Bridge Types Tested - Normal Floor Position. Model Scale 1 : 40

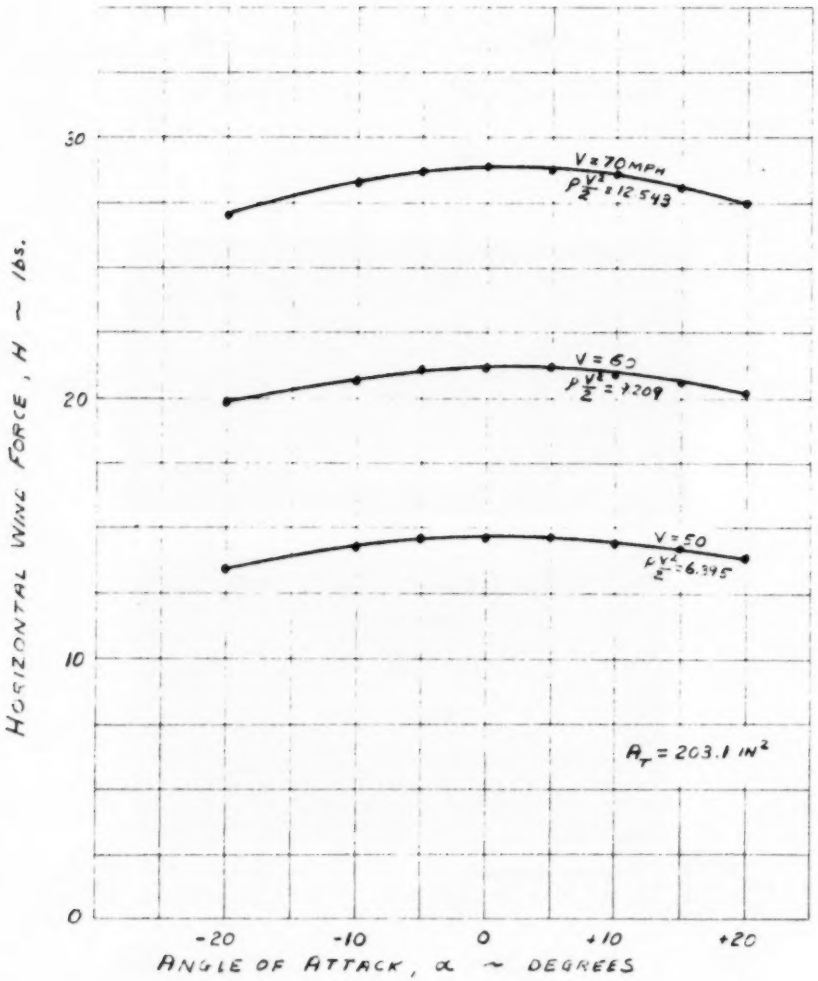


FIG. 6. Horizontal Wind Forces on Single Truss with Variable Wind Velocity and Angle of Attack.

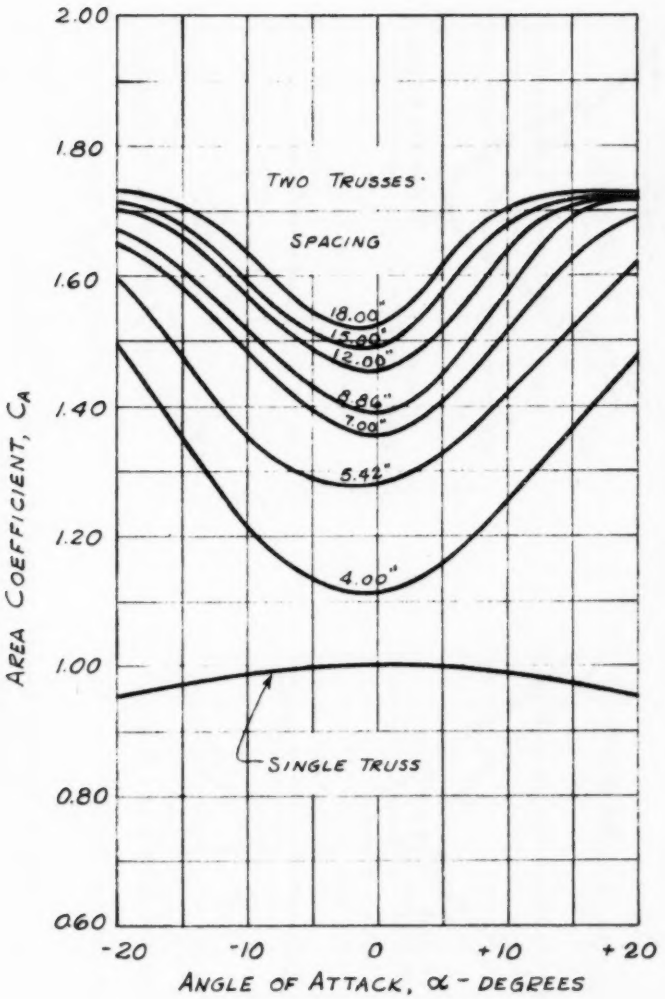


FIG. 7. Area Coefficients for Single Truss and Two Trusses at Various Spacings.

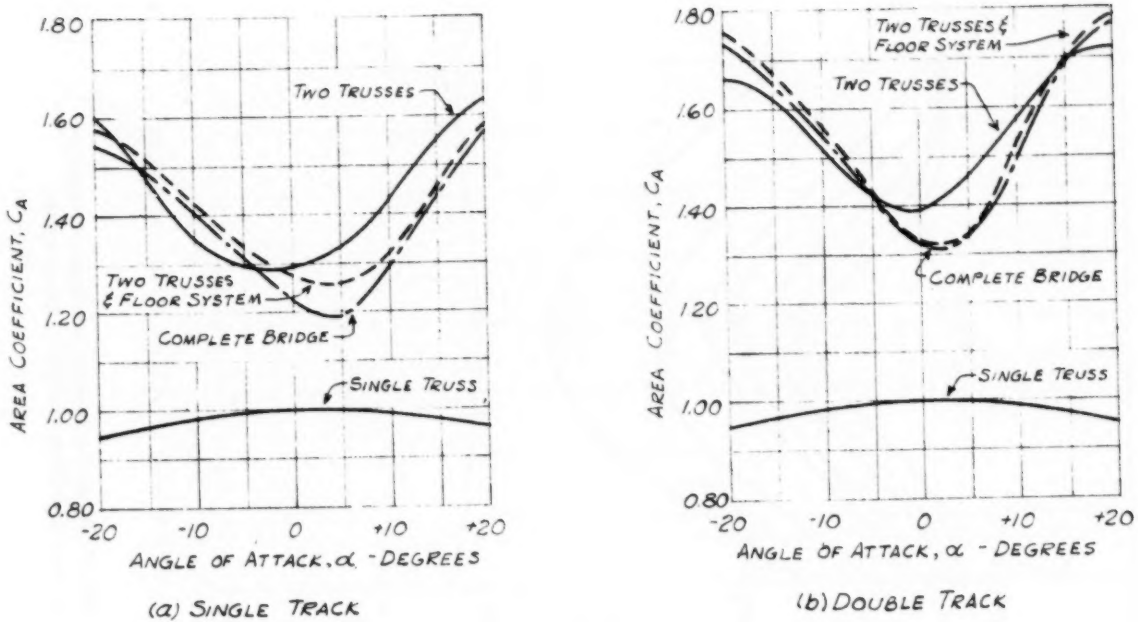


FIG. 8. Railroad Bridges. Effect of Successive Addition of Components on Area Coefficient.

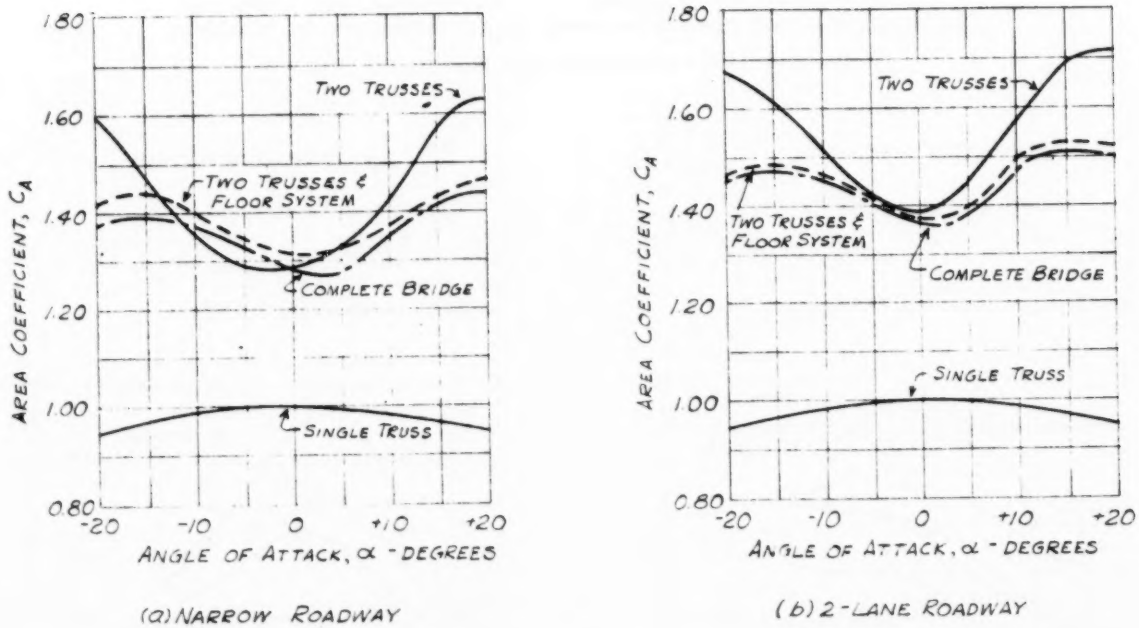


FIG. 9. Highway Bridges. Effect of Successive Addition of Components on Area Coefficient.

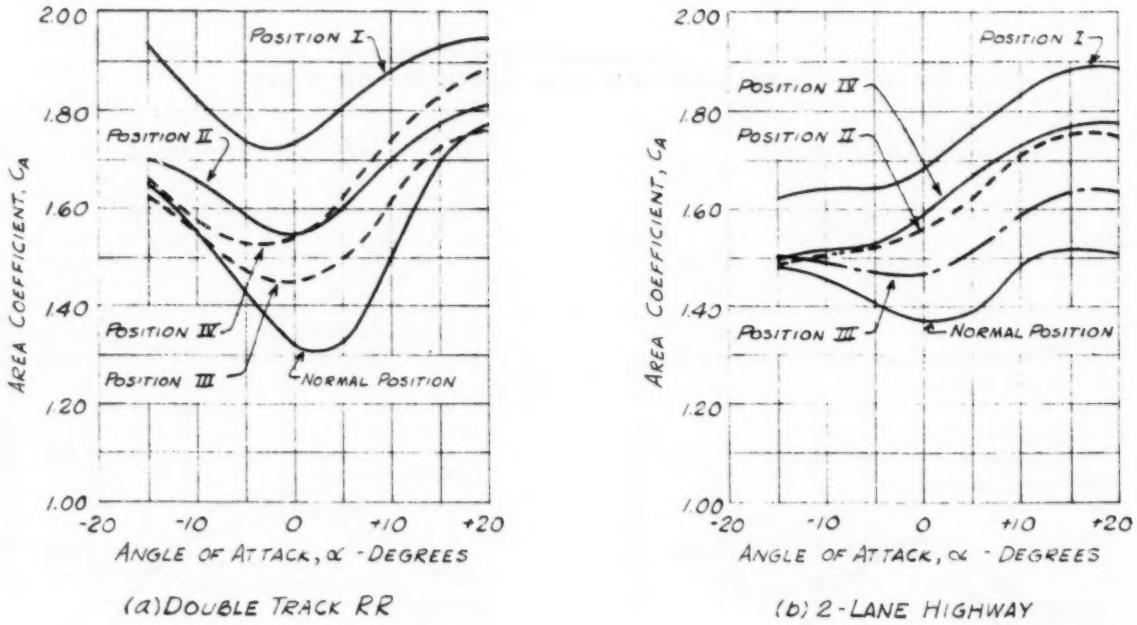


FIG. 10. Effect of Floor Position on Area Coefficient.

In Normal Position Bottom of Floor Beam Is Level with Bottom of Truss Chord

Position	I	3.5" Below Normal	Position	III	0.9" Above Normal
"	II	2.0" " "	"	IV	2.5" " "

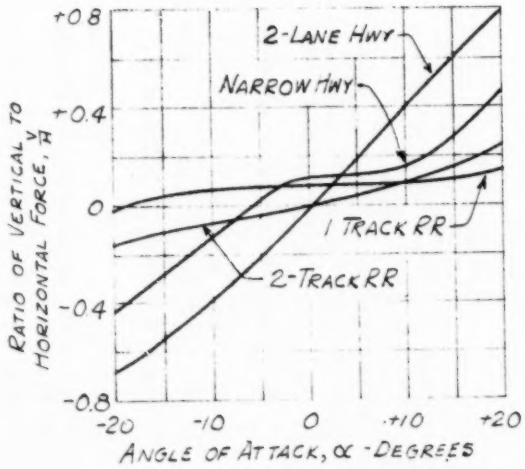


FIG. 11. Ratio of Vertical Lift Force to Horizontal Force.

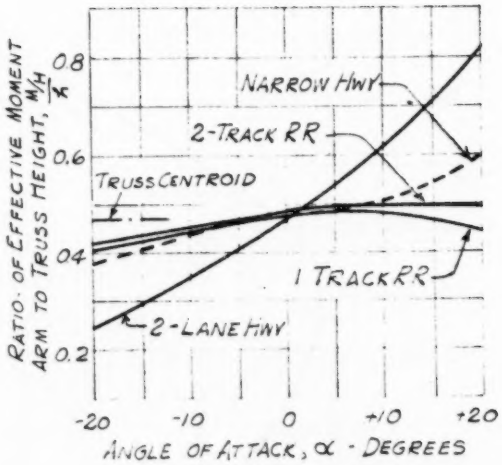


FIG. 12. Effective Overturning Moment Arm-Ratio to Truss Depth.

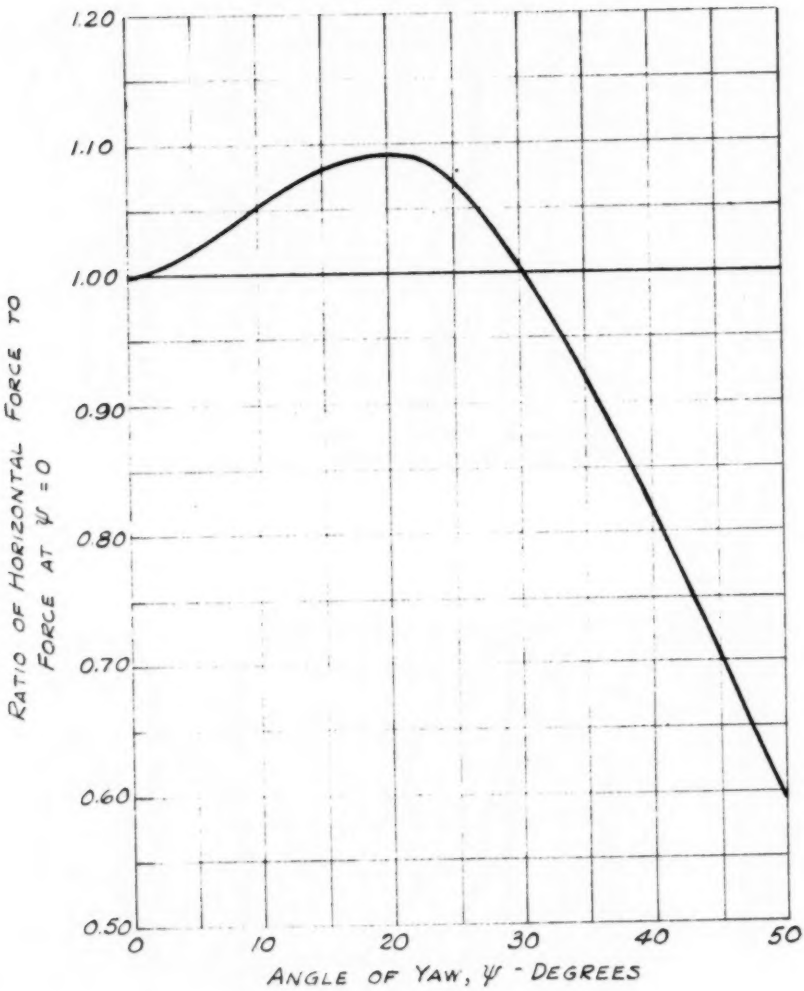


FIG. 13. Effect of Yaw on Horizontal Force-Ratio to Force at $\psi = 0$

TABLE I

Position	Double Track RR ($\alpha = 15^\circ$)			Two-Lane Hwy ($\alpha = 15^\circ$)		
	Due to Floor ΔC_A	Effective Floor Area sq. ins.	Unshielded Area sq. ins.	Due to Floor ΔC_A	Effective Floor Area sq. ins.	Unshielded Area sq. ins.
I	0.25	51	82.5	0.19	39	91.6
II	0.09	18	82.5	0.07	14	91.6
Normal	0.01	2	30.8	-0.16	-33	38.7
III	0.04	8	71.8	-0.04	- 8	79.7
IV	0.15	30	71.8	0.08	16	79.7

TABLE II

	Experimental (109.5 mph)			Specifications (AREA & AASHO) (50 lbs. per sq. ft.)		
	Trusses	Net Effect of Floor lbs.	Total lbs.	Trusses	Net Effect of Floor lbs.	Total lbs.
Normal Floor Position						
2-Track RR	125	1	126	141	-7	134
1-Track RR	107	3	110	141	-7	134
2-Lane Hwy	125	-12	113	106	20	126
Narrow Hwy	115	-9	106	106	20	126
Floor Position IV						
2-Track RR	125	11	136	141	21	162
2-Lane Hwy	125	6	131	106	42	148

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